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CHAPTER 6.   STREETS, INLETS, AND STORM DRAINS

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1.0   OVERVIEW

1.1   General

This chapter contains design information and criteria for the design of urban stormwater collection and conveyance systems. This chapter specifies criteria for: 1) the allowable drainage encroachment within public streets, 2) allowable types, sizing, and placement of inlets, and 3) sizing and design of storm drains.

The design procedures presented in this chapter are based upon fundamental hydrologic and hydraulic design concepts. Use of this chapter requires an understanding of the Rational Method for runoff hydrology (Chapter 5, Calculation of Runoff), open channel hydraulics (Chapter 8, Open Channels), and pipe hydraulics.

1.2   Design Storms

The storm water system, at a minimum, shall commence at the point where the 2-year flow rate for fully developed conditions equals or exceeds 5 cubic feet per second (cfs) or street spread/depth limitations are exceeded. The following criteria apply to the design of streets, inlets, and storm drains:

- The 2-year flow rate shall be calculated for the fully developed conditions to show that street inundation criteria are met.

- The hydraulic grade line (HGL) for the 25-year flow shall not exceed the elevation of the gutter for a pipe system and shall not exceed channel capacity with a freeboard of 6 inches for an open channel.

- All street storm drains and channels shall be designed to convey the 25-year flow with fully developed conditions.

- Inlet, storm drain and channel capacity shall be designed so that the HGL from the 100-year flow under fully developed conditions meets street inundation criteria and all runoff is directed to the receiving detention basin or drainage system.

In determining peak flow rates for design events for the design of streets, inlets, and storm drains, the effects of detention basins on adjacent properties may be accounted for if assurances that detention will be provided for perpetuity are provided and maintenance is assured. Assurances must typically be provided through recorded documents with land use restrictions and maintenance responsibilities properly noted.

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2.0   STREET DRAINAGE

### 2.1   Allowable Inundation Depth and Spread

The purpose of limitations for inundation depth and spread is to ensure the safe travel of the public and emergency vehicle access. These limitations are based on the use classification of the street. The criteria used for inundation limitations are the minor storm event (2-year) and the major storm event (100-year). These criteria govern the design of the longitudinal street slope, inlet size, and inlet spacing. Table ST-1 presents city street inundation criteria. For collector and arterial streets, no flow shall pass through an intersection for a 25-year or lesser event.

Table ST-1
City of Springfield Street Inundation Criteria

Street Classification	Minor Storm (2-year)	Major Storm (100-year)
Local	• No flow from one side of street to another • No curb overtopping • Inlets spaced for a maximum spread of 8 feet from curb face with one clear 10-foot travel lane	• No curb overtopping
Collector	• No flow from one side of street to another • No curb overtopping • Inlets spaced for a maximum spread of 8 feet from curb face with one 12-foot travel lane clear for non-residential collectors, and two 10-foot travel lanes clear for non-residential collectors	• No curb overtopping
Arterial	• No flow from one side of street to another • No curb overtopping • For secondary arterials, inlets spaced for a maximum 12-foot spread with two 12-foot travel lanes open. • For other arterials, inlets spaced for a maximum 6-foot spread with one full lane and one 8-foot lane clear in both directions	• No curb overtopping

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2.2   Curb and Gutter Hydraulics

The required design standard for public streets includes curb and gutter. Complete standards for design and construction of streets and curb and gutter can be found in the *City of Springfield, Missouri Design Standards for Public Improvements* (Design Standards). The minimum allowable longitudinal slope for a public street is 0.5 percent. The maximum allowable longitudinal slope for a public street ranges from 5 to 10 percent, depending on street classification. Standard gutter width is 2 feet, with a depth of 2 inches, resulting in a gutter cross slope, S_w, of 0.0833 foot per foot (ft/ft). The standard cross slope, S_x, for a public street is ¼ inch per foot or approximately 2.1 percent. Gutter depression, a, in Figure ST-1, is 1.5 inches for typical Springfield street sections. A greater longitudinal slope results in higher velocities, less spread, and generally, higher inlet efficiencies. Figure ST-1 shows a typical curb and gutter cross-section, with variables that are used in equations in subsequent sections.
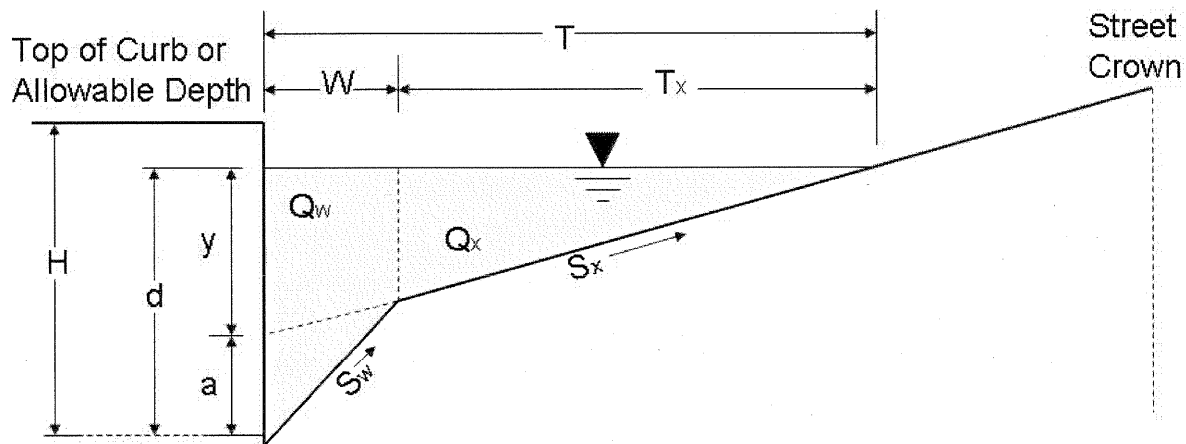
For Springfield:

S_w = 0.0833 ft/ft, S_x = 0.021 ft/ft, a = 1.5 in

Figure ST-1
Typical Curb and Gutter Cross-Section

The street, curb and gutter, and inlets must be designed simultaneously to meet the street inundation criteria found in Section 2.1. Gutter flow and spread, T, can be determined using Manning's equation for open channel hydraulics (Chapter 8, Open Channels). Assuming the standard gutter and street cross-section found in the Design Standards, gutter flow can be determined by using Figure ST-2, Standard Gutter Spread.

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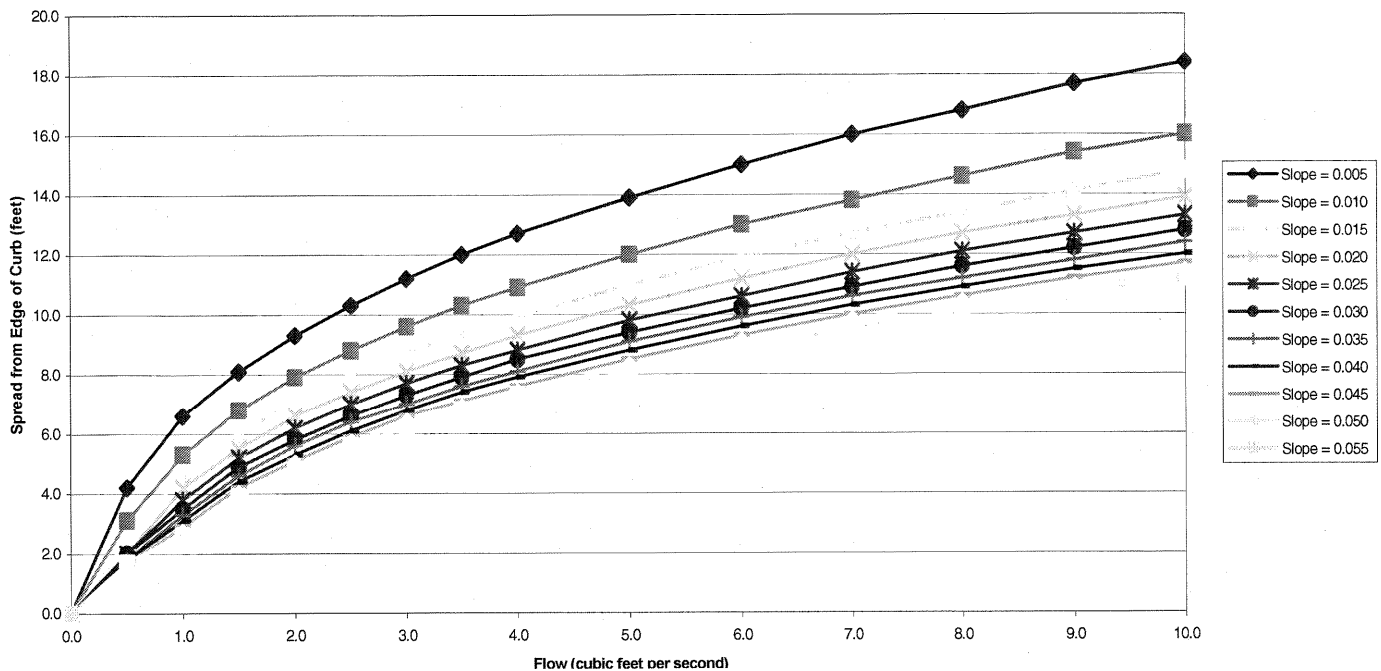
Figure ST-2
Standard Gutter Spread

2.3   Roadside Swales

Springfield's standard for public streets in new developments is a curb and gutter section with enclosed storm drains. Other street sections, such as roadside swale sections or other Low Impact Development (LID) designs, will be considered on a case-by-case basis. A subdivision variance will be required through the subdivision approval process, and generally, the developer or property owners association will be required to maintain all street rights-of-way and LID components. Alternative designs shall take into consideration topography and the proposed development and must adhere to the principles of containing the 100-year flow within the right-of-way or drainage easement and preventing erosion. Refer to Chapter 8, Open Channels, and Chapter 10, Water Quality, for more information on open channel hydraulics, grass swales, and LID techniques.

3.0   INLETS

3.1   General

Properly designed inlets are vital components of the urban stormwater collection and conveyance system. Inlets collect excess stormwater from the street, transition the flow into storm drains, and can provide maintenance access to the storm drain system. Inlets can be made of cast iron, steel, concrete, and/or

pre-cast concrete and are installed on the edge of the street adjacent to the street gutter or in the bottom of a swale.

Roadway geometry often dictates the location of pavement drainage inlets. In general, inlets are placed at all low points (sumps or sags) in the gutter grade and on the upstream side of intersections, median breaks, pedestrian ramps, and crosswalks. The spacing of inlets placed in addition to those required by geometric controls is governed by the allowable inundation criteria given in Section 2.1.

Inlets shall not have a vertical opening greater than 6 inches. If a larger opening is used, a steel bar with a minimum diameter of ¾ inch shall be placed in the opening so there is no vertical opening greater than 6 inches.

3.2   Inlet Types

3.2.1   In Public Streets

There are three main inlet types per the *City of Springfield, Missouri Standard Drawing Details for Public Improvements* (Standard Drawing Details) for use in a public street:

- SS-5—Curb opening inlet for 24-inch diameter or larger storm drain with 6-inch × 7-foot opening.

- SS-6—Curb opening inlet for 18-inch diameter or smaller storm drain with 6-inch × 7-foot opening.

- SS-6-R—Recessed curb opening inlet with 9-inch × 7-foot opening with a ¾-inch bar across middle of opening. This is the standard inlet for collector and arterial streets, but is often not suitable for residential areas with many driveways.

Variations of SS-6 include:

- SS-6G—Standard SS-6 inlet with a vaned grate.

- SS-6S—Short version of a standard SS-6 with 6-inch x 3-foot opening (4-foot-square top).

Other inlet types and combinations include grate inlets, recessed curb opening inlets with grates, and 4-foot curb opening inlets that are recessed and/or have a grate. Vaned grate inlets are permissible if reviewed and approved by the City, particularly in redevelopment areas where utilities, decorative sidewalk, planter trees, signs, etc., may prevent the practical placement of inlets and storm drains behind the curb. All grate inlets shall be bicycle safe.

3.2.2   Outside of Public Streets

Inlet types used outside of a public street include:

- Open Sided Drop Inlet or Area Inlet—Typically a 4-foot or larger square, concrete, off-street inlet, with openings on any or all sides to collect water in a ditch or collect water in an area sump.

- Grate Area Inlet—Typically used to collect runoff in a parking lot. May be used in grass areas with less than ¼ acre of drainage area due to clogging potential.

3.3   Hydraulic Evaluation

The hydraulic capacity of an inlet is dependent on the type of inlet (e.g., SS-5, SS-6, SS-6-R, or other) and the physical characteristics of the location (e.g., continuous grade or sump conditions, street cross slope, gutter geometry). The methodology for determining the hydraulic capacity of the standard inlets for continuous grade and sump conditions is described in Sections 3.3.1 and 3.3.2, respectively. The following general criteria apply when selecting inlet locations:

- Inlets must be located to limit gutter flows to no more than 5 cfs for the 2-year event and to meet spread limitations in Table ST-1.

- Inlets are required at all low points (sumps or sags) in the gutter grade and on the upstream side of intersections, median breaks, pedestrian ramps, and crosswalks. Flow cannot be directed across a street.

3.3.1   Curb Opening Inlets on a Continuous Grade

The inlet and storm drain system shall begin at or before the location where the 2-year flow equals 5 cfs. The capture efficiency of a curb opening inlet is dependent on the length of the opening, the depth of flow at the curb, the street cross slope and the longitudinal gutter slope. Even the most efficient inlet design will often result in some inlet bypass; therefore, the inlet efficiency must be calculated so that the capture and bypass for each inlet can be determined. Many variables used in the equations below are shown in Figure ST-1. The efficiency, E, of a curb opening inlet on a continuous grade is calculated as:

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$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$
         Equation ST-1

for $L < L_T$, otherwise $E = 1.0$

In which:

L = design curb opening length (ft) (standard inlets are 7- and 3-ft lengths)

L_T = curb opening length required to capture 100 percent of gutter flow (ft)

To factor in gutter depression or for a curb opening inlet that is not recessed:

$$L_T = 0.6 Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_x}\right)^{0.6}$$
         Equation ST-2

In which:

Q = gutter flow (cfs)

S_L = longitudinal street slope (ft/ft)

S_x = street cross slope (ft/ft)

n = Manning's roughness coefficient (typically 0.016 for a street cross-section with concrete curb and gutter and asphalt paving—see Chapter 8, Open Channels for Manning's n values for other surfaces)

To factor in gutter depression or for a recessed curb opening inlet:

$$L_T = 0.6 Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_e}\right)^{0.6}$$
         Equation ST-3

The equivalent cross-slope, S_e, can be determined from:

$$S_e = S_x + \frac{a}{W} E_0$$
         Equation ST-4

In which:

a = gutter depression (ft)

W = width of gutter below breakpoint in cross slope (recessed section width) (ft)

E_0 = ratio of flow in the recessed section of the gutter to total gutter flow determined by Manning's equation

$$E_0 = \cfrac{1}{1 + \cfrac{S_w/S_x}{\left[1 + \cfrac{S_w/S_x}{(T/W)-1}\right]^{8/3} - 1}}$$

<div align="right">Equation ST-5</div>

In which:

S_w = gutter cross slope (ft/ft)

T = spread from edge of curb (ft)

A 10 percent clogging factor is assumed for a single curb opening inlet.

Example:

Given a longitudinal street slope of 3 percent and a design flow rate of 5 cfs, determine the efficiency of a single SS-5 curb opening inlet on grade. Determine the flow captured by the inlet and the carryover flow to the next inlet.

Based on the street longitudinal slope of 3 percent (S_L = 0.030 ft/ft) and a flow rate of 5 cfs, the spread, T, can be determined from Figure ST-2 to be 9.4 feet.

For standard city streets the gutter cross slope, S_w, is 0.0833 ft/ft, the street cross slope, S_x, is 0.021 ft/ft, and the gutter width, W, is 2 feet. E_0 can be calculated using Equation ST-5, as follows:

$$E_0 = \cfrac{1}{1 + \cfrac{0.0833/0.021}{\left[1 + \cfrac{0.0833/0.021}{(9.4/2)-1}\right]^{8/3} - 1}}$$

$E_0 = 0.60$

Given a gutter depression, a, of 1.5 inches (0.125 feet), the equivalent slope, S_e, can be calculated using Equation ST-4:

$$S_e = 0.21 + \left(\frac{0.125}{2} \cdot 0.60\right)$$

$S_e = 0.0585$ ft/ft

L_T, the total curb opening length required to capture the entire 5 cfs, can then be calculated from Equation ST-3 using a Manning's n = 0.016 for the concrete and pavement gutter and street cross-section:

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$$L_T = 0.6 \cdot 5^{0.42} \cdot 0.03^{0.3} \left(\frac{1}{0.016 \cdot 0.0585} \right)^{0.6}$$

$L_T = 27.0$ feet

The efficiency of a single SS-5 curb opening inlet ($L = 7$ ft) can be calculated as follows:

$$E = 1 - \left(1 - \frac{7}{27} \right)^{1.8}$$

$E = 0.42$ or 42%

Therefore, without accounting for clogging, a single SS-5 inlet would be expected to capture about 42 percent of the flow, or 2.1 cfs, in this example. After applying a 10 percent clogging factor for a single curb opening inlet, the inlet would be expected to capture 1.89 cfs, with an actual efficiency of approximately 38 percent. Additional inlets would be required to deal with the carryover of 3.11 cfs (5.0 cfs – 1.89 cfs captured by the single SS-5 inlet).

3.3.2   Inlets Located in Sumps

Inlets must be placed in all street sumps. No street cross slope that directs drainage across a street at a sump is acceptable. Clogging of inlets in a sump can lead to hazardous driving conditions. Therefore, grate inlets alone should not be used in a sump. Single grate inlets must assume a 50 percent clogging factor.

Inlets in sumps function like weirs for shallow depths, but as the depth of stormwater increases, they begin to function like an orifice. The Federal Highway Administration (FHWA) (1979) provides guidance on the transition region based on significant testing. The hydraulic capacity of a curb opening inlet or a vaned grate inlet operating as a weir is expressed as:

$$Q_i = C_w L_w d^{1.5}$$

Equation ST-6

In which:

Q_i = inlet capacity (cfs)

C_w = weir discharge coefficient

L_w = weir length (ft), length of inlet opening acting as weir

d = flow depth (ft)

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Table ST-2 provides values of C_w and L_w for various inlet types. Note that the expressions given for curb opening inlets without depression should be used for recessed curb opening inlets if L is greater than 12 feet.

The hydraulic capacity of a curb opening inlet or a vane grate inlet operating as an orifice is expressed as:

$$Q_i = C_o A_o \sqrt{2gd}$$
<div align="right">Equation ST-7</div>

In which:

C_o = orifice coefficient

A_o = orifice area (square feet [ft^2])

g = gravitational acceleration (32.2 ft/s^2)

d = characteristic depth (ft) defined in Table ST-2

Values for A_o and C_o are provided in Table ST-2. The weir and orifice equations provided above are also applicable for other types of inlets (e.g., combinations, slots, etc.); however, coefficients for these other inlet types may vary from coefficients in Table ST-2.

Table ST-2
Sump Inlet Discharge Variables and Coefficients
(Modified from Akan and Houghtalen 2002)

Weir Inlet Types	C_w	L_w[1]	Weir Equation Valid For	Definition of Terms
Curb opening inlet (SS-5, SS-6)	3.00	L	$d < h$	L = Length of curb opening h = Height of curb opening $h = d_i - (h/2)$ d_i = Depth of water at curb opening
Recessed curb opening inlet (SS-6-R)[2]	2.30	$L + 1.8W$	$d < h + a$	W = Lateral width of recessed section a = Depth of curb depression
Vane Grate Inlet	3.00	$L + 2W$	$d < 1.79(A_o/L_w)$	A_o = Clear opening area W = Width of grate
Orifice Inlet Types	C_o	A_o[3]	**Orifice Equation Valid For**	**Definition of Terms**
Curb opening inlet (SS-5, SS-6) or recessed curb opening inlet (SS-6-R)[2]	0.67	hL	$d_i > 1.4h$	d_i = Depth of water at curb opening $d = d_i - (h/2)$ h = Height of curb opening
Vane Grate Inlet	0.67	Clear opening area	$d > 1.79(A_o/L_w)$	d = Depth of water over grate A_o = Clear opening area

[1] The weir length (L_w) should be reduced where clogging is expected.
[2] If L > 12 feet, use the expressions for curb opening inlets that are not recessed.
[3] The orifice area (A_o) should be reduced where clogging is expected.

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3.3.3 Inlet Clogging

The effects of inlet clogging must be taken into account. A clogging factor must be selected based on the debris and trash condition on the street. During a storm event, street inlets are often loaded with debris by the first-flush runoff volume. To account for the resulting loss in capacity, a 50 percent reduction in capacity shall be used for a single grate inlet and a 10 percent reduction in capacity shall be used for a single curb opening inlet. Multiple inlet units are generally not acceptable in new construction; however, in such cases, a smaller reduction in total inlet capacity may be acceptable.

3.3.4 Plan and Calculation Submittals

It is important to carefully track runoff flow rates, bypassing flow rates, flow spreads, and other parameters related to gutter flow and inlet capacity for all design storms. Refer to Chapter 4, Plan Submittal, for submittal requirements, report formats, and submittal checklists.

4.0 STORM DRAINS

4.1 General

Storm drains convey runoff collected by the inlet system. The storm drain system includes inlets, pipes, channels, junctions, bends, outlets, and other appurtenances.

4.1.1 Horizontal Alignment

The following criteria apply to the horizontal alignment of storm drains:

- The horizontal alignment of storm drains shall be behind the curb unless site conditions or street geometry make it necessary to be under the street. The number of street crossings should be minimized.

- Storm drain alignment shall be straight with bends only at junction boxes. See Section 4.1.4 for cases where bends may be allowed.

- Public storm drains outside of the right-of-way must be located within a drainage easement. Table ST-3 shows minimum easement widths. Additional width may be required based on depth of the storm drain and site-specific conditions. Refer to Chapter 12, Easements and Maintenance, for more information.

- Horizontal clearance between storm pipes and sanitary sewer and water lines shall be a minimum of 10 feet. Refer to the Design Standards for complete sanitary sewer separation requirements and contact the appropriate utility for water line separation requirements.

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Table ST-3
Minimum Easement Widths Required for Storm Drains

Inside Horizontal Dimension (inches)	Minimum Easement Width (ft)
12 – 18	10
21 – 36	15
42 – 72	20
Larger	To be determined

4.1.2   Vertical Alignment

The following criteria apply to the vertical alignment of storm drains:

- The storm drain shall have a design velocity range from a minimum of 2 ft/s for the 2-year event up to a maximum of 15 ft/s for the 25-year event.

- Vertical clearance between storm drains and sanitary sewer and water lines shall be 18 inches. In cases where 18 inches of vertical clearance cannot be attained, refer to the Design Standards for complete sanitary sewer separation requirements and contact the appropriate utility for water line separation requirements.

- The minimum cover for storm drains is 12 inches and must be sufficient to withstand American Association of State Highway and Transportation Officials (AASHTO) HS-20 loading on pipe. Additional cover may be required according to manufacturer requirements.

- Vertical drops greater than 6 feet require a special design that addresses the need for structure protection and energy dissipation.

4.1.3   Pipes

Allowable pipe types and required standards for various pipe materials are listed in Table ST-4. Acceptable pipe materials include reinforced concrete, metal and plastic. Reinforced concrete pipe shall be used under the street and when any part of the pipe is within 2 feet of the street back of curb or in other locations where it is not feasible to use metal or plastic.

The minimum allowable pipe diameter in a public system is 15 inches or its equivalent in elliptical pipe. The minimum allowable slope is 0.5 percent, and the maximum allowable slope is 15 percent, unless hydraulic conditions govern. When a pipe slope in excess of 15 percent is necessary, the pipe must be anchored and stabilized to ensure it will not fail. See Section 4.2 for additional information on hydraulics of storm drains.

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Table ST-4
Allowable Pipe Types

Material	Abbreviation	Standard
Reinforced Concrete Round Pipe	RCP	ASTM C-76
Reinforced Concrete Elliptical Pipe	RCEP	ASTM C-507
Reinforced Concrete Pipe Arch	RCPA	ASTM C-506
Precast Concrete Flared End Sections	FES	ASTM C-76
Corrugated, Galvanized Steel Round Pipe	CMP	ASTM A-760 AASHTO M-36
Corrugated, Galvanized Steel Pipe Arch	CMPA	AASHTO M-167
Galvanized Steel Flared End Sections	FES	ASTM A-760
Corrugated Polyethylene Pipe	CPP	ASTM F2306
Cast-in-place Reinforced Concrete Box Culverts	RCB	MDOT Specification
Precast Concrete Box Culvert	RCB	ASTM C-1433
Smooth-walled Pipe (High Density Polyethylene)	HDPE	ASTM-F894
Ribbed Metal Pipe	RMP	Not specified

4.1.4   Junction Boxes

The primary functions of junction boxes include:

- Providing maintenance access.

- Providing ventilation.

- Serving as junctions when two or more pipes merge.

- Providing flow transitions for changes in pipe size, slope, and alignment.

The following criteria apply to bends and junctions:

- A standard junction box is required at all horizontal bends, grade changes, and changes in pipe size, shape, or material. Exceptions may be granted for pipes with a diameter greater than 36 inches that have other access within 300 feet.

- A bend within 3 feet of a junction or inlet box with access may be used to achieve a more perpendicular junction. In this case, the angle for pipes 24 inches and smaller may not exceed 60 degrees, and the angle for pipes larger than 24 inches may not exceed 30 degrees.

- The distance between access points to the storm drain system shall not exceed 300 feet.

- Junction boxes shall be constructed as shown on SS-1 in the Standard Drawing Details.

4.1.5   Bends and Transitions

In cases where bends and transitions are acceptable, it may be necessary to provide gradual transitions in size or alignment to minimize energy losses and potential for blockage. When bends are necessary, bends at pipe joints should generally not exceed 2 degrees or otherwise cause unacceptable head losses in the system. Transitions should be gradual and shall not decrease the size of the pipe in the downstream direction. Special structures that control head losses to an acceptable level may be necessary for other transitions.

4.1.6   Outlets

Outlet structures are transitions from pipe flow into open channel flow or still water (e.g., ponds, lakes, etc.). The primary function of these structures is to minimize erosion in the receiving channel or water body. Generally, outlets from small pipes consist of riprap and shall extend a minimum of 5 times the pipe diameter downstream of the pipe and have a width of 3 times the pipe diameter. The riprap pad should be depressed 6 inches at the outfall. A standard detail for a riprap outlet is provided in the Standard Drawing Details. Refer to Chapter 7, Bridges and Culverts, and Chapter 8, Open Channels, for more information on outlet structures and riprap design.

4.1.7   Other Appurtenances

Examples of other specialized appurtenances include flow splitters and deflectors and flap gates. Flow splitters separate incoming flow and send it in two or more directions. Flow deflectors minimize energy losses in manholes, junction chambers, and flow splitters. Flap gates are placed on outlets to prevent backflow in areas subject to high tailwater or flood flow. These are structures that require individual consideration and specialized design prior to City acceptance.

4.1.8   General Safety Requirements

The following safety criteria apply to all public storm drainage systems:

- Headwalls and wingwalls associated with storm drain outfalls shall include guardrails, handrails, or fencing in conformance with highway design safety standards and applicable building codes.

- Handrails shall be required in all areas where the drop from the headwall or wingwall exceeds 30 inches. See the Design Standards for handrail design standards. Refer to AASHTO standards for safety rails along designated bikeways.

- Trash racks shall be provided at all entrances to storm drains.

4.2   Storm Drain Hydraulics

4.2.1   General

The design of a storm drain system requires data including topography, drainage boundaries, soil types, and locations of any existing storm drains, inlets, and junction boxes. Identification of the type and location of utilities to identify potential conflicts must be completed early in the design process. The drainage area to each inlet shall be identified and the design flows calculated to determine the necessary pipe capacity.

Once the storm drain system is designed, the hydraulic capacity shall be evaluated for the design storms using energy grade line (EGL) calculations starting at the downstream terminus of the system. Hydraulic grade line (HGL) calculations shall be performed for the design storms. All transitions, bends, entrance and exit conditions, tailwater conditions, and other losses shall be accounted for in HGL calculations.

4.2.2   Design Storm and Storm Drain Capacity

All storm drains shall be designed to convey the 25-year flow with the HGL no higher than the gutter of the street, and the HGL for the 100-year flow no higher than the top of curb. Spread inundation criteria shall be met for the 5- and 100-year flows, and all runoff shall be directed to the downstream detention and/or drainage system appropriately. The allowable velocity range for pipes and concrete channels shall be a minimum of 2 ft/s for the 2-year event up to a maximum of 15 ft/s for the 25-year event. The initial check of pipe or channel capacity shall be determined using Manning's equation. Figure ST-3 relates depth of flow and pipe diameter to hydraulic parameters for circular pipes and may be used as a tool for sizing. Final design of pipes and channels may require further hydraulic analysis. Refer to Chapter 8, Open Channels, for additional information on the application of Manning's equation.

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Hydraulic Elements $\dfrac{V}{V_f}, \dfrac{Q}{Q_f}, \dfrac{A}{A_f}$ and $\dfrac{R}{R_f}$
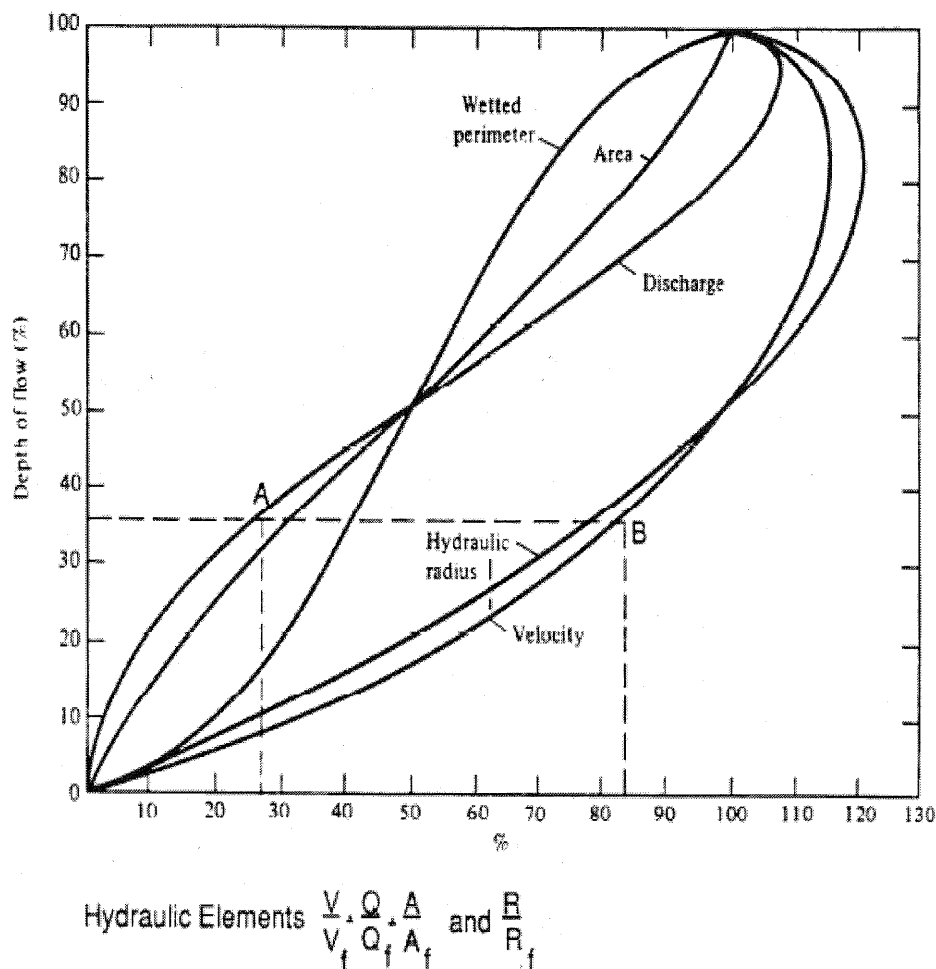
Figure ST-3
Hydraulic Characteristics of Circular Conduits
(Chow 1959)

4.2.3   Hydraulic and Energy Grade Line Calculations

HGL calculations are required for all storm drain designs. The EGL is calculated using the principle of conservation of energy and the energy equation for open channel or pressure flow as follows:

For open channel flow:

$$z_1 + d_1 + \frac{V_1^2}{2g} = z_2 + d_2 + \frac{V_2^2}{2g} + h_L$$

Equation ST-8

For pressure flow:

$$z_1 + \frac{p_1}{\gamma} + \frac{V_1^2}{2g} = z_2 + \frac{p_2}{\gamma} + \frac{V_2^2}{2g} + h_L$$

Equation ST-9

Where:

z = elevation (gravity) head (ft)

d = depth of flow (ft)

V = velocity of flow (ft/s) = Flow (Q) / Area (A)

p/γ = pressure head (ft)

γ = unit weight of water (lbs/ft^3)

g = gravitational acceleration (32.2 ft/s^2)

h_L = head loss (ft) = $h_f + \Sigma\, h_m$

Where:

h_f = head loss due to friction (ft) from Equation ST-10

h_m = minor head loss (ft)

The HGL is calculated by subtracting the velocity head, $V^2/2g$, from the EGL. When the velocity is 0, the HGL and EGL are the same.

Head losses must be accounted for in the design of storm drains in order to calculate the HGL. The following sections describe methods for calculating head losses due to pipe friction, junction box and manhole structures, pipe junctions, transitions (contractions and expansions), and system outfalls (exit losses). Akan and Houghtalen (2003) provide a concise overview of head losses in storm drain systems, and the following sections are based upon their reference, which incorporates procedures developed by FHWA (1996). Thorough documentation of loss calculations, input parameter guidance, software tools, and example calculations are available from the FHWA (1996).

4.2.3.1   Head Loss Due to Pipe Friction

The friction loss, h_f, due to flow resistance in a storm sewer is calculated as:

$h_f = LS_f$

In which:

> L = length of the sewer pipe (ft)

> S_f = friction slope (ft/ft), typically taken as the bottom slope of the pipe, S_0, as a simplifying assumption for partially full, gravity flow storm sewers.

In addition to the energy losses due to friction in storm sewers, losses occur at transitions, bends, junctions, and other appurtenances. Many of the equations for estimating these losses have been determined empirically, and some variations should be expected based on site specific conditions. The following sections present general guidance for calculating energy losses for various storm sewer components.

4.2.3.2   Inlet and Manhole Losses

The energy loss encountered by flow in an incoming storm sewer is determined differently depending on whether the water in the manhole or junction submerges the incoming pipe invert. If the invert is above the water surface in the manhole, then the flow in the incoming pipe will not be affected by junction losses, and the outflow pipe is treated like a culvert (refer to Chapter 8, Bridges and Culverts). If the invert of an inflow pipe is below the water surface in a manhole, the head loss, h_{LM}, is:

$$h_{LM} = K_0 C_D C_d C_Q C_p C_B \frac{V_o^2}{2g}$$

In which:

> K_0 = initial loss coefficient

> V_o = velocity in the outflow pipe (ft/s)

> g = gravitational acceleration (32.2 ft/s^2)

> C_D, C_d, C_q, C_p, and C_B = correction factors for pipe sizes (D), flow depth (d), relative flow (q), plunging flow (p), and benching (B) (For more information, see Akan and Houghtalen 2003, FHWA 1996, and FHWA 1994.)

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The initial loss coefficient for an incoming pipe is calculated using the following equation:

$$K_0 = 0.1\left(\frac{b}{D_o}\right)(1 - \sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \qquad \text{Equation ST-12}$$

In which:

b = manhole or junction diameter (ft)

D_o = outflow pipe diameter (ft)

θ = the angle between the inflow and outflow pipes

The correction factor for pipe size, C_D, is significant only for high-pressure flow and is a function of the outflow pipe diameter, D_o, and the inflow pipe diameter, D_i:

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad \text{Equation ST-13}$$

This equation is used only when d_{aho}, the flow depth in the manhole or junction above the outlet pipe invert, is >3.2D_o. Otherwise, $C_D = 1.0$. The HGL elevation at the upstream end of the outflowing pipe can be used as an approximation of d_{aho}.

The correction factor for flow depth, C_d, is significant only for low-pressure or free surface flow. It is determined using the following equation:

$$C_d = 0.5\left(\frac{d_{aho}}{D_o}\right)^{0.6} \qquad \text{Equation ST-14}$$

This equation should only be used when d_{aho}<3.2D_o. Otherwise, $C_d = 1.0$.

When three or more pipes enter the structure at approximately the same elevation, the correction factor for relative flow, C_Q, is calculated using the following equation:

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad \text{Equation ST-15}$$

In which:

Q_i = flow rate of inflow pipe (cfs)

Q_o = flow rate of outflow pipe (cfs)

This equation is used if $h_p > d_{aho}$, where h_p = distance from the flow line of the higher elevation inlet pipe to the center of the outflow pipe. It is also used if a higher elevation flow plunges into an access hole that has both an inflow pipe and an outflow pipe in the bottom of the access hole. For other cases, $C_p = 1.0$.

The correction factor for benching, C_B, is 1.0 for flat benching configurations, which are typically used in Springfield. See Akan and Houghtalen (2003) for correction factors if other benching configurations are used. Pipe Junctions

Where a lateral pipe is connected to a larger trunk pipe without the use of a manhole, the head loss due to the junction, h_{Lj}, is calculated as follows:

$$h_{Lj} = \frac{Q_o V_o - Q_i V_i - Q_L V_L \cos \theta}{0.5g(A_o + A_i)} + \frac{V_i^2}{2g} - \frac{V_o^2}{2g} \qquad \text{Equation ST-16}$$

In which:

V_i, V_o and V_L = velocities in the inflow, outflow and lateral pipes, respectively (ft/s)

Q_i, Q_o and Q_L = flow rates in the inflow, outflow and lateral pipes, respectively (cfs)

A_i and A_o = cross-sectional area of flow in the inflow and outflow pipes, respectively (ft^2)

θ = angle between the inflow and outflow pipes

4.2.3.3   Transitions

The energy loss due to a gradual pipe expansion, h_{LE}, is calculated as follows:

$$h_{LE} = K_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \qquad \text{Equation ST-17}$$

In which:

K_e = expansion loss coefficient from Table ST-5

V_1 = velocity upstream of the expansion (ft/s)

V_2 = velocity downstream of the expansion (ft/s)

The energy loss due to a gradual pipe contraction, h_{LC}, is calculated as follows:

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$$h_{LC} = K_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right)$$

Equation ST-18

In which:

K_c = contraction loss coefficient

V_1 = velocity upstream of the expansion (ft/s)

V_2 = velocity downstream of the expansion (ft/s)

In general, $K_c = 0.5K_e$.

Table ST-5
Typical Values for K_e for Gradual Enlargement of Pipes in Nonpressure Flow

D₂/D₁	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.140	0.86	1.02	1.06	1.04	1.00

Source: After FHWA (1996).

4.2.3.4   Bend Loss

The energy loss due to a bend, h_{Lb}, is calculated as follows:

$$h_{Lb} = 0.003\Delta \frac{V^2}{2g}$$

Equation ST-19

In which:

V = velocity in bend

Δ = angle of curvature in degrees

4.2.3.5   Exit Losses

The energy loss at a storm sewer system outlet, h_{LO}, is calculated using the following equation:

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$$h_{LO} = \frac{V_o^2}{2g} - \frac{V_d^2}{2g}$$

In which:

V_o = velocity in the outlet pipe

V_d = velocity in the downstream channel

When a storm sewer discharges into a reservoir, $V_d = 0$. For discharge to an open channel, Manning's equation can be used to estimate, V_d, based on channel geometry and roughness.

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5.0   REFERENCES

Akan, A.O. and R.J. Houghtalen. 2003. *Urban Hydrology, Hydraulics and Stormwater Quality Engineering Applications and Computer Modeling.* Hoboken, N.J.: John Wiley and Sons.

Chow, V.T. 1959. *Open-Channel Hydraulics.* New York, Toronto, London. McGraw-Hill.

City of Springfield. 2002. *City of Springfield, Missouri, Standard Drawing Details for Public Improvements.* May 10.

City of Springfield. 2002. *City of Springfield, Missouri, Design Standards for Public Improvements.* May 10.

U.S. Federal Highway Administration (FHWA). 1996. *Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22.* (FHWA-SA-96-078). Washington, D.C.: FHWA.

———. 1994. *Energy Losses through Junction Manholes, Volume 2.* (FHWA-RD-94-081). Washington, DC: FHWA.

———. 1979. *Design of Urban Highway Drainage.* (FHWA-TS-79-225). Washington, DC: FHWA.

———. 1972. *Hydraulic Design of Improved Inlets and Culverts, Hydrologic Engineering Circular 13.* (FHWA-EPD-86-109). Washington, DC: FHWA.